Merced River
Robinson/Gallo Project - Ratzlaff Reach
Engineering Report

California Department of Water Resources
San Joaquin District
River Management Section

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Project Partners:

California Department of Water Resources
California Department of Fish and Game
Delta Pumping Plant Fish Protection (4-Pumps) Agreement
U.S. Fish and Wildlife Service / Anadromous Fish Restoration Program
U.S. Bureau of Reclamation / CALFED Bay-Delta Program

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Introduction

The Merced River has undergone extensive modification over the years to provide agricultural and municipal water supply, flood control, and power generation, as well as raw materials such as gravel products and gold. As early as the 1870's, large canal systems were built to divert Merced River water for agricultural use. Several dams were built to regulate flows, the largest being New Exchequer Dam (completed in 1967) which can store up to 1,032,000 acre-feet of water in its reservoir. Mining for gold and aggregate downstream of the dams has been extensive, leaving tailings and numerous pits within the river corridor.

The manipulation of the river has led to loss and degradation of natural habitat. There have been several impacts to salmon in particular. As a result of dam construction, access to historic upstream spawning grounds has been lost and the availability of coarse sediment has been reduced. The reduction in coarse sediment has resulted in decreased quantity and quality of spawning and rearing habitat. In addition, the large ponds left by in-stream mining create both habitat for warmwater predator fish species which prey upon juvenile salmon, and barriers for coarse sediment migration.

Flow regulation has led to reduced peak flows and an overall reduction in the average flow in the river, which has contributed to a general narrowing of the channel (Vick, 1995). The two-year flow event prior to regulation (pre-Exchequer) was approximately 16,000 cfs at Exchequer gage. Flow records show that since New Exchequer Dam began operation, the two year event is approximately 2,300 cfs at Snelling gage (Figure 1). This means that the high flows which traditionally scoured and flushed fines and vegetation from active gravel bars and banks and delivered coarse sediment are all but absent. As a result

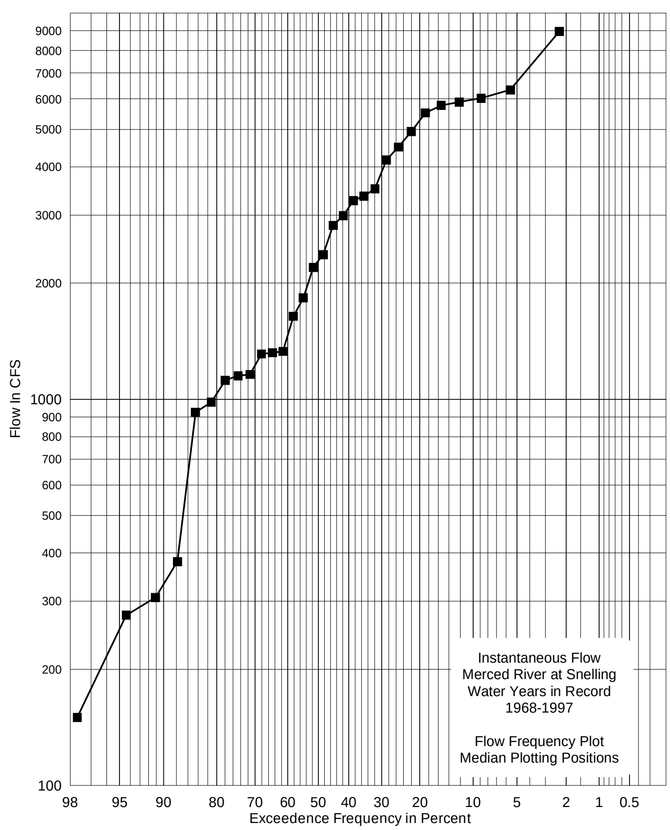

Figure 1

there is encroachment of vegetation and abandonment of alluvial deposits, which leads to narrowing and armoring of the channel.

A loss of gravel recruitment to the lower reaches of the river can also be attributed to dams. The river is "sediment starved" during higher flows, and tends to recruit sediment from channel banks and beds. This can cause bed coarsening and channel incision, which when combined with an overall reduced flow can further narrow the channel and lead to abandoned floodplains.

The major problems associated with the project reach were twofold - biological and geomorphological. First, the large, deep pond provided excellent habitat for warmwater predators of young salmon. With the river flowing through the pond, smolts were easy targets for the predators. The second problem was that the condition of the reach provided no possibility for natural functioning of the river. Sediment could not be transported past that point since velocities were slowed to near stagnation during normal bankfull events.

Condition of Project Reach

This phase of the project is located on the Merced River between river miles 40.0 and 40.5, approximately 6 miles southwest of the town of Snelling (Figure 2). It is Phase I (Ratzlaff Reach) of the 3.8 mile Robinson/Gallo Restoration Project (Figure 3).

The Robinson/Gallo project is characterized by miles of gravel pits created in the last forty years. The pits were excavated to a depth of fifteen to twenty feet, or about ten feet below the current low water level in some areas. At that level the mining operation encountered a thick layer of clay. The berms which once separated the gravel pits from the river have been reduced over the years to low islands along the old river channel.

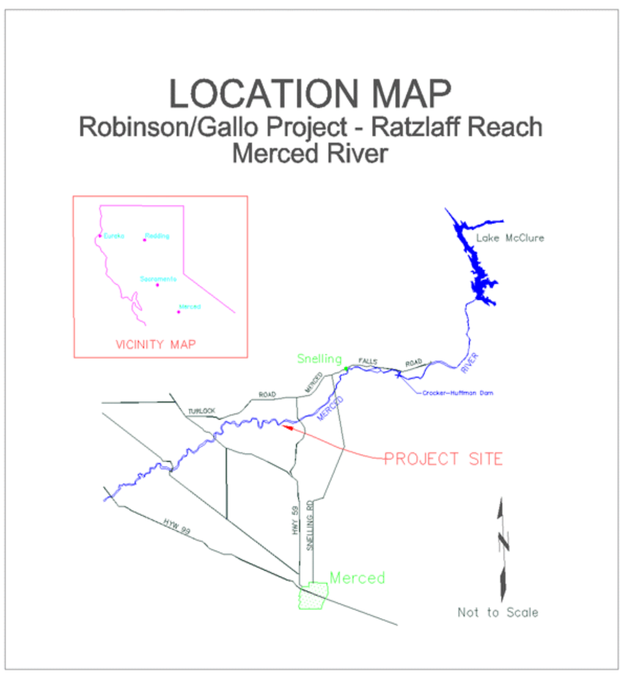

Figure 2

Until early 1997, the Robinson/Gallo Project reach still had one functioning berm in the Robinson reach, but it failed due to sustained high flows that January.

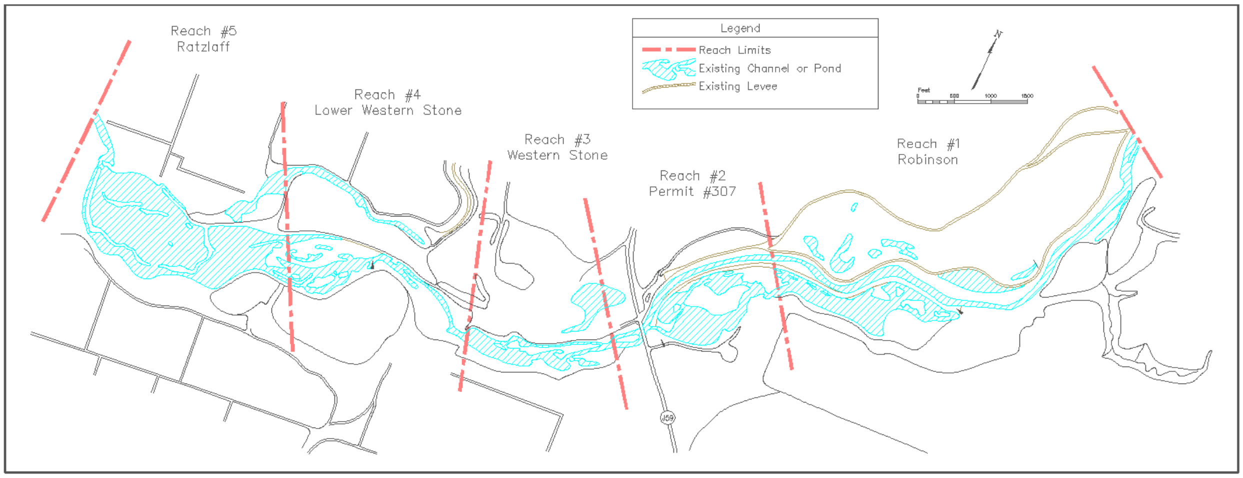

Figure 3 - Robinson/Gallo Project Overview

Failure of berms in the Ratzlaff Reach of the project had allowed the river to flow through an abandoned gravel pit. The river abandoned a channel that was already heavily constricted and overgrown with vegetation. The berms had limited the river width to fifty feet in some areas before it failed.

Goals and Objectives

The Robinson/Gallo project will be designed and constructed in several phases over several years. The different phases will be designed using the same methodologies and with the entire project in mind to ensure that they are compatible. The goal is to have a continuous and functional river for the entire 3.8 miles.

The goal for the Ratzlaff Reach of the project was to isolate a large pond from the river channel and to build the channel into a more natural, functional reach, which would be beneficial to the salmon of the Merced River. Objectives include:

1. Eliminate and/or isolate juvenile salmon predator habitat.
2. Increase the quantity and quality of spawning habitat for chinook salmon.
3. Increase the quantity and quality of rearing habitat for chinook salmon.

4. Improve river and flood plain dynamics by reconfiguring the channel to better conform with the present flow regime.
5. Create and enhance the riparian corridor.
6. Create a more natural stream and improve sustainability.

Particle Size Distribution of Planned Fill Material

Before developing a set of typical channel and flood plain dimensions to be used in the design, the maximum allowable depths at selected flows must be established through particle movement calculations. This requires an estimate of the particle size distribution for the constructed channel. The two potential sources of fill material identified were in the same geographic area. They are located on Highway J59 approximately six miles north of the project. With the close proximity of the two sites, sampling of just one of them was sufficient, and the material at the Blasingame site was chosen to be sampled because it is a daily operated site and was easier to access and sample.

Eight samples were taken at the Blasingame site in December of 1997. Each one consisted of a five gallon bucket filled with material which was excavated from one of several locations chosen to represent the overall composition of the tailings. The samples were brought back to the lab for size distribution analysis.

A sieve analysis was done for each sample separately, and all were graphed together along with the mean for the group (see Figure 4). The analysis shows that the average D_{84} was approximately 170mm, and the D_{50} is 61mm. These values were used in designing the dimensions for the project, since virtually the entire design channel was to be constructed with this material.

A reasonable range of sizes for the fill material was developed using the data gathered. The guideline can be seen graphically in Figure 5. The ranges are as follows: 0%-3% passing 6mm, 1%-10% passing 16mm, 4%-37% passing 32mm, 44%-86% passing 100mm, 100% passing 300mm.

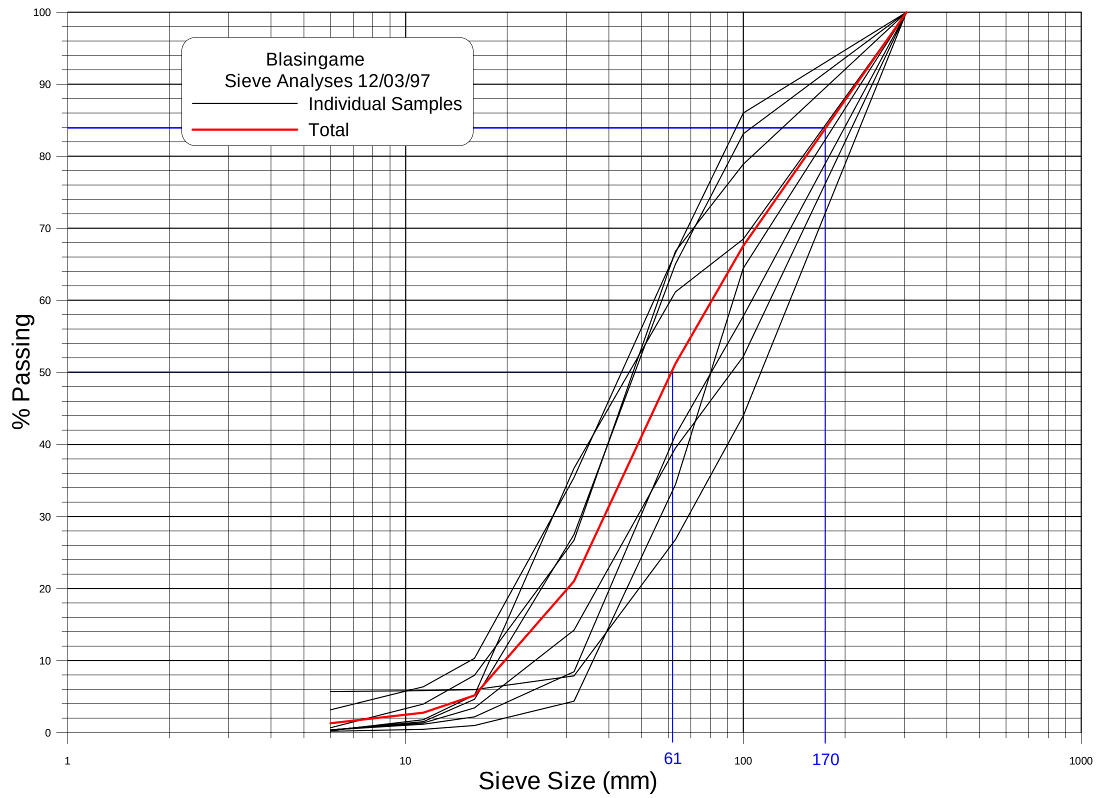

Figure 4

Prior to beginning construction, the material chosen to be used as fill was sampled by Kleinfelder, Inc. for size distribution in April of 1999. The material was located at the Calaveras Materials Inc. "La Grange Pit" approximately two miles from the Blasingame site. Sieve analysis of 3,840 lbs of the material resulted in a distribution as seen in Figure 6. The D_{50} and D_{84} are nearly identical to that of the Blasingame analysis from 1997.

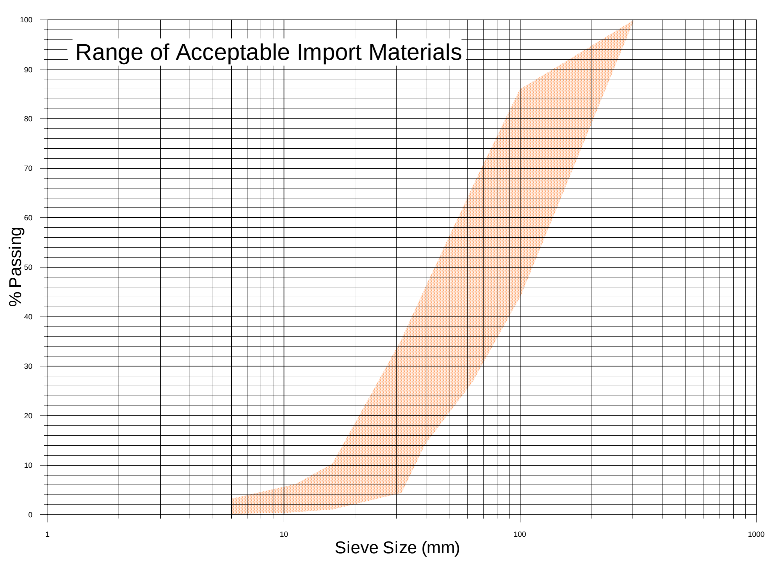

Figure 5

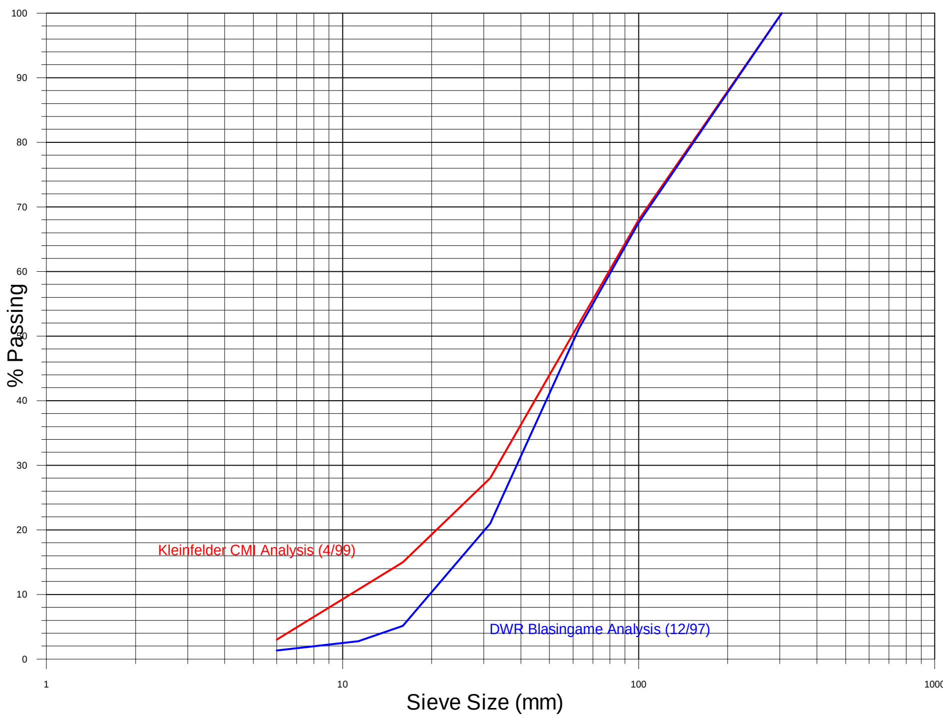

Figure 6 - Blasingame/CMI Comparison

During construction of the project, samples were taken to assess whether the material being imported was meeting the specifications set forth. A set of four samples weighing a total of 1,140 lbs were taken from the flood plain material in August, 1999. While the material appeared to have more fines than previously anticipated, the D_{50} and D_{84} were close enough to the guidelines to be acceptable (see Figure 7).

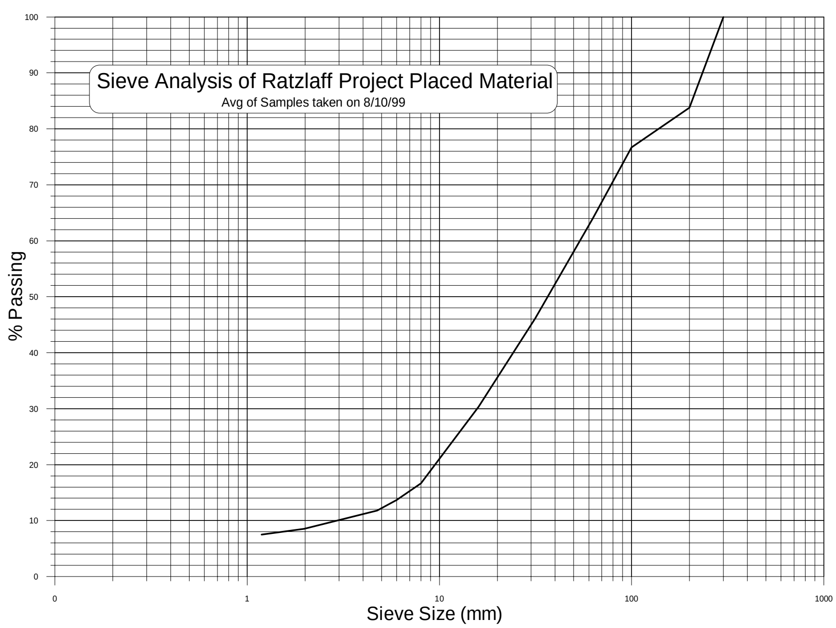

Figure 7

Bankfull Discharge Determination

The bankfull discharge is one of the most important parameters to determine before the design sections can be developed. This is because it is regarded as the discharge most influential in forming the channel. In alluvium rivers, bed load is transported at flows smaller than bankfull and is increased as bankfull flow is approached. Once bankfull is exceeded, flow spills onto the flood plain, which reduces the rate of increase in bed load transport.

A report by Trinity Fisheries Consulting (Laird and others, 1989) which was prepared for the California Dept. Of Fish and Game, contained an estimate for the bankfull discharge. The estimate of 3,000 to 3,500 cfs was not accompanied by any data or statement of methodology, but a sample cross-section of the bankfull channel was provided. Using this section, calculations were done using Chezy-Manning Equation with established values for “n” in order to estimate the amount of flow the section would be capable of carrying. According to the given dimensions, the results indicate that the channel would be expected to carry more than 4,200 cfs. This flow, according to Figures 1 and 8, would have an exceedence frequency of 28 to 35 percent, or a return period of approximately 2.9 to 3.6 years (Return period = 1/frequency). DWR staff decided to investigate the river to determine the bankfull discharge at the project reach using stream gages and bankfull formations.

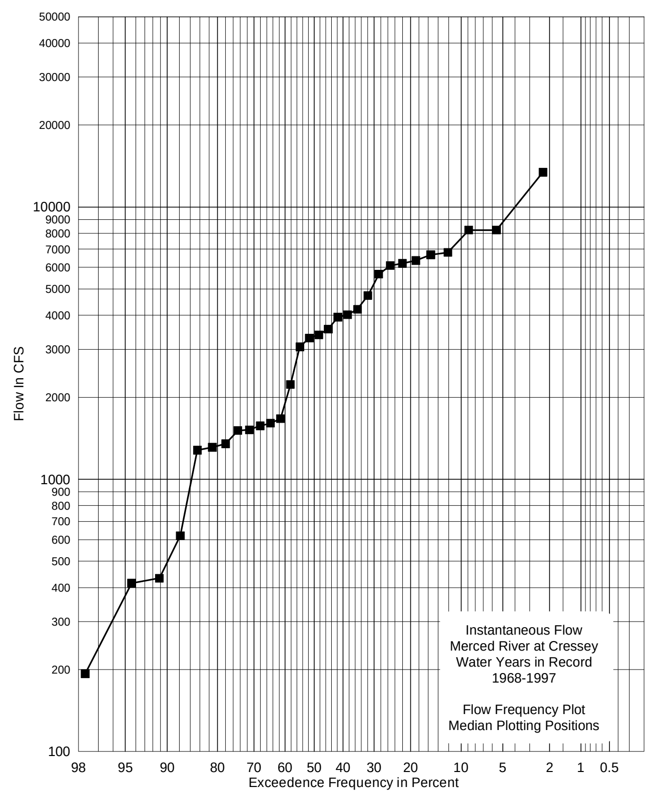

Figure 8

The bankfull discharge should occur near a return period of 1.5 years, but may be up to a 2.5 year flow (Leopold, 1994). The method for determining bankfull discharge in a stream involves using a well established stream gage with a long history of data collection. The only two gaging stations in the vicinity of the project were Snelling, approximately 4.5 river miles upstream, and Cressey, approximately 14 river miles downstream of the site. Staff obtained gage data for both gages including rating tables and shifts. Data taken after the high flows of January 1997 were used in determining current shifts because of possible changes in channel geometry.

The gaging stations were visited for an on-site determination of the bankfull discharge. The designers recognize that true “bankfull discharge” indicators do not exist on a regulated stream. Keeping that in mind, it is still desirable to have frequent bed movement on the designed project. A bankfull flow would therefore need to be selected that occurs on a frequency of 1.5 to 2.5 years.

Recognizing this, we visited two gaging stations: one at Cressey and one at Snelling. Using the most current rating tables, staff gage readings were taken from the tables at the corresponding 1.5 and 2.5 year flows. These elevations were staked out at the gaging stations. Vegetation, breaks in slope, and particle size were all observed in an attempt to determine the “bankfull flow” at these sites. A likely elevation for the bankfull discharge was chosen from these indicators.

At the Snelling gage, two observations were found to fall within the 1.5 to 2.5 year gage heights. The observations were averaged to determine that the bankfull discharge for the reach is 1,420 cfs. At the Cressey gage site, there were also two observations that fell within the prescribed range. The average of these came to 2,000 cfs. According to the flow-frequency curves for the gages, both were approximately 1.66 year events (Figures 1, 8). The fact that the return period is similar for both strongly suggests that this is the proper bankfull return period. Factoring in local runoff and the location of the project site in relation to the two gaging stations warrants adjusting the flow to 1,700 cfs for the project.

Development of Typical Sections

Three typical sections were designed for the project: a riffle section, a run section and a pool section. Each of these were designed with three important flows in mind. The first is the spawning flow (225 cfs), which is the anticipated minimum release for this reach during the spawning period. The second important flow is the bankfull flow (1,700 cfs), which was determined in the bankfull flow study illustrated above. The third flow is the flood flow, which for design purposes was determined to be 8,000 cfs, or approximately the 25 year event (Figure 1).

The flood flow was chosen based on the January 1997 releases from New Exchequer Dam. This choice was confirmed in several discussions with the Army Corps of Engineers and the Merced Irrigation District, and it reflects the feeling that the design flow of the river will not likely be raised above that value. There are several reasons for this assumption. The impacts of raising the design flood flow on the river much above the existing 6,000 cfs would be extensive. Many levees would breach, resulting in capture of numerous gravel pits by the river and subsequent loss of habitat. Also, low lying communities, power lines, sewer plants, and bridges on the Merced River and the lower San Joaquin River could be affected. Flooding impacts would also likely be felt on the Stanislaus and Tuolumne Rivers, as well as at the Delta. In the 1997 high flows, New Exchequer Dam and its reservoir functioned well in what is considered to be a 100 year event by releasing 8,000 cfs. The dam released 6,000 to 8,000 cfs for nearly two months, including several days at 8,000 cfs, with moderate impacts. The highest flow recorded at the Snelling gage since New Exchequer Dam began operation in 1967 was 8,970 cfs, which occurred in January 1997. The next highest flow recorded was in January of 1983, when the gage showed 6,330 cfs.

The water surface slopes used in the design depended on both the type of section and the flow. For flood flows in all sections, a slope of 0.0014, or 0.14% was used. This slope determination was made by combining a photogrammetric survey of the project, which showed water surface elevations at a known discharge, with a topographic survey of the upstream end of the project after the high flows in 1997. An average slope for the reach could be calculated using this data. At very high flows, irregularities in slope seen at low flows are attenuated and an average slope is observed for extended reaches. At low flows, which for our purposes apply to bankfull flows and lower, the slopes depend on the type of section. In a riffle section, the slope is 0.002, or 0.2%, to attain the proper velocities for spawning. In a paper by the U.S. Fish and Wildlife Service, observations of the Merced River showed an optimum depth for salmon spawning to be greater than 1.9 feet, and the optimum velocity to be between 1.3 and 2.1 ft/sec (USFWS, 1997). Calculations show that at a slope of 0.002, the depth is 2 feet and the velocity is 2.14 ft/sec using a 50 foot channel width. In both the run and pool sections, the slope is reduced to 0.001 (0.1%) or less so that velocities are lower.

The values for “n” used in the Chezy-Manning equation were estimated based on experience and several studies done on the subject. In the calculation for bankfull flow, a value of 0.030 was used because most of the underwater surface will be gravel. In the flood flow calculation, the value was raised to 0.040 to account for flood plain vegetation.

Using both the Chezy-Manning equation and the surface roughness equation, dimensions were determined for each type of design section. The surface roughness equation,

$$v = (gHs)^{1/2}(5.65\log(H/D_{84})+2.83)$$

is used to determine depth at spawning flows, since it does not take into account the effect of vegetation on roughness. The Manning equation is used for higher flows. Necessary dimensions for the channel such as depth, width, and side slopes were estimated using this method. This process also led to the determination of the height of the berm which will surround the pond. The berm was designed using these calculations to contain up to 8,000 cfs in the channel. Above this flow, the river can rise two feet without overtopping the berm except at two engineered notches called “equalization saddles” (Figure 9). These saddles will act as designed low spots in the berm to allow overtopping and filling of the pond with minimal damage to the berm. Hydraulic calculations show that the berm itself will not be overtopped

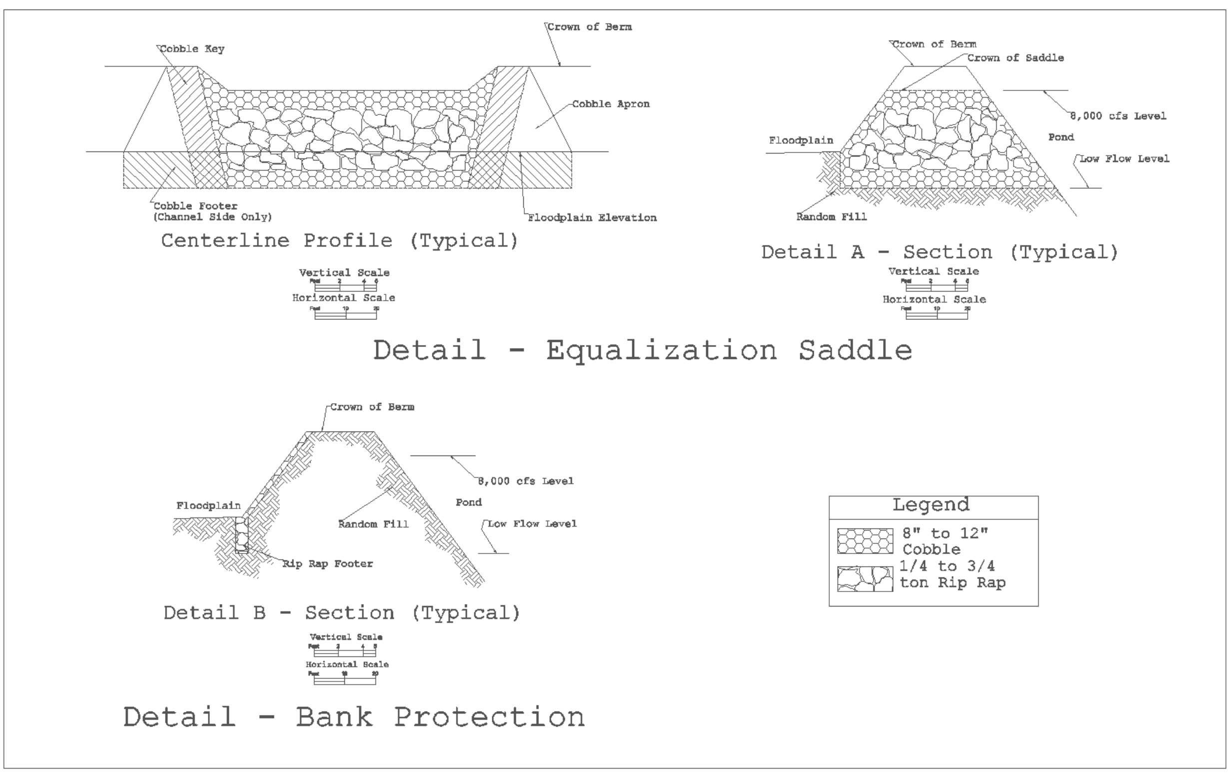

Figure 9 - Equalization Saddles

until the flow in the river reaches approximately 12,600 cfs.

Several checks were done on the typical design sections to assure that dimensions were adequate with respect to current knowledge. Width/depth ratio, entrenchment, and critical depth were all checked for the design. Using Rosgen's classification system, this stream type is a C3 to C4. The width/depth ratio for this stream type is greater than 12 and entrenchment ratio is greater than 2.2. Calculating these values for the design sections yields more than 26 for the width/depth ratio for all sections and 2.28 for the entrenchment ratio. The entrenchment ratio is determined by dividing the channel width at twice the bankfull height by the bankfull channel width (Rosgen, 1996). The critical depth refers to the depth at which the shear stress on particles exceeds the critical shear stress, and movement occurs. Normally bed movement or failure should occur at a flow near the designated bankfull flow, but since there are little or no gravel recruitment possibilities in this reach of the river it is acceptable for the critical depth to be higher. This would allow the reach to remain relatively stable in the near term. There will be opportunities for the systematic addition of spawning gravels to the riffle reaches of the project in the future. Having a relatively stable base will help ensure that the overall geography of the reach will remain after construction until other phases of the project can be implemented. Critical depth is calculated using the Shields Criterion, and an equation derived from the criterion:

$$H_c = \frac{(1.65)(\tau_c^*)(D_{84})}{S}$$

gives the critical depth. When the value of τ_c^* is 0.02 the resulting critical depth is 13.1 feet. When substituting the D_{50} particle (61 mm) for the D_{84} particle in the calculation above, the critical depth is 4.7 feet, which is very near bankfull flow in the riffle sections. The value of τ_c^* in this calculation is also 0.02, although normally a higher value would be used for the D_{50} in a stream where the bed has been placed and graded by natural processes. Since the material will be placed artificially, it will not have the structural characteristics that naturally placed gravel would have. Over time the new material will be moved and graded by the river and as a result the τ_c^* will likely rise. The design riffle section has a depth of five feet at bankfull (1,700 cfs), which may be considered to be the point at which the D_{50} particle will move.

When the Andrews relationship (Andrews 1994, p2247) for the dimensionless shear stress, τ_c^*, is applied using the analyses illustrated in the Particle Size Distribution section, the resulting values are lower than 0.02 for the D_{84} particle. According to Andrews, the lowest applicable value for τ_c* should be 0.02, therefore, the value used in the critical depth calculations above was correct, assuming the assumption made about the D_{50} was accurate.

Design Methodology

The channel characteristics such as depth and width at the various flows were arrived at as outlined above. Other characteristics, such as side slopes, sinuosity, and radius of curvature were also important considerations in the process. Usually, the overall sinuosity and mean radius of curvature is determined based upon the known or expected bankfull discharge and the current flow regime.

The bankfull width of 118 feet was determined based on calculations done to determine required widths at low flows, and by observations of nearby reference reaches. As was stated in the Bankfull Discharge Determination section of this paper, the Trinity Fisheries report gave an estimate of the bankfull discharge that was higher than our investigation showed. In the paper, the bankfull width was stated as approximately 205 feet. Since their bankfull flow was larger than the results of our analysis, we felt that their width was probably larger than it should be. The preliminary bankfull dimensions were found by designing a bankfull section that would carry the flow, and would attain the necessary depths at spawning flow. Using section 24+00 from the particle movement calculation as a reference, the width at a flow of 1,700 cfs is about 128 feet. This section is not in a perfect reference reach, but is acceptable because it is in a fairly straight section of channel recently reformed by high flows that contributed gravel from the stream banks.

The values for the factors mentioned above may be found using the methodology outlined in Rosgen (1996). The sinuosity (or ratio of channel length to valley length) for this classification should be greater than 1.4. The sinuosity of the design is 1.1 when taking the valley length as the distance along the centerline of the floodway. The meander wavelength is determined by the relation $L = 10.9w^{1.01}$ (Rosgen, 1996), where w is the bankfull width. Using this equation, the meander wavelength should be around 1,350 feet. In designing the channel, existing land forms were allowed to influence the geometry which resulted in a meander wavelength of an acceptable 1,100 feet. The mean radius of curvature is usually 2.5 to 3 times the bankfull width.

Using this guideline, the radius should be about 300 to 350 feet. As stated before, existing land forms were allowed to guide the design geometry, and as a result the radius of the curves in the design range from 250 feet to 450 feet. This allows for a more diverse and natural looking stream while keeping the mean radius of curvature near the guideline. Designing with these guidelines in mind, and using fill material which is large enough to withstand moderate to large flows before significant movement occurs, should ensure a quasi-stable and self-maintaining reach of river.

The hydraulics of the final design were checked by the creation of a hydraulic model using the HEC-RAS modeling program. The results showed that depths were adequate at the design flows and that the flood plain begins to be inundated when the discharge exceeds the bankfull flow of 1,700 cfs.

Construction

Work began on the construction phase of the project on June 21st, 1999. After several days of access road preparation the contractor began work on the project itself. The construction began at the upstream end of the project and proceeded from there station by station. Generally, the strategy was to roughly fill in the footprint of the design up to approximately three feet below final flood plain grade and above the existing water line. In several areas a large amount of unconsolidated silts and sands were encountered on the pond bottom, which could potentially result in an unstable base for the fill material. The contractor was required to remove the unconsolidated material before placement of fill, and as a result the project required approximately 40% more random fill material than originally estimated. In spite of this, the Department of Water Resources was able to keep the contractor on a time line close to the estimated schedule. The Department of Fish and Game's 1600 permits required all work to cease in the wetted portion of the channel by October 1st to allow passage of migrating Chinook salmon, and that requirement was met. Work continued on dry parts of the project after that date, but all work was completed by mid-October.

There were several minor modifications to the original plan. First, the upper end was constructed from a point somewhat downstream of the original start point of station 1+00.

This was done because of concerns for an agricultural pump near station 1+00 on the left bank. Future phases of the project will address and resolve issues related to this pump. Next, two elderberry bushes were discovered shortly prior to construction on the existing berm at approximately station 23+00 to 25+00 of the project. The original design called for this section of berm to be removed, but since the plants had not been included in the mitigation plan the design channel was rerouted to avoid them. As a result, the channel was moved from 5 feet to 15 feet to the right (facing downstream) at stations 21+00 to 27+00, and a mound was left with the bushes that is approximately three feet higher than the design flood plain.

Another modification to the original plan was a small berm that was added to separate most of the backwater area from the main channel. The berm's crown is at flood plain elevation and was added to address sediment transportability concerns as expressed by Jennifer Vick of Stillwater Ecosystem, Watershed and Riverine Sciences. The final modification to the plan was to the bank protection. The original plan called for bank protection on the berm from station 14+00 to 17+00, and on the right bank from station 28+50 to 31+00. The bank protection on the berm was reduced by about fifty percent and the protection at the end of the project was extended by approximately 200 feet. For further information see Appendix D, the construction report prepared by DWR's Division of Engineering.

Prior to bidding, the Department estimated the project would cost approximately $3.7 million to construct. This included purchase of all imported material and placement. The final cost of construction was $3.36 million, which leaves approximately $340,000 of the funded amount for maintenance of the project. The project was constructed under budget even though it required a significantly larger amount of fill material than originally estimated.

Geomorphic Monitoring

Monitoring of this project will be for both morphological and biological processes. The morphological components of the project will be monitored by the Department of Water Resources, and are outlined here. Biological monitoring will be done by the Department of Fish and Game, and will not be discussed here.

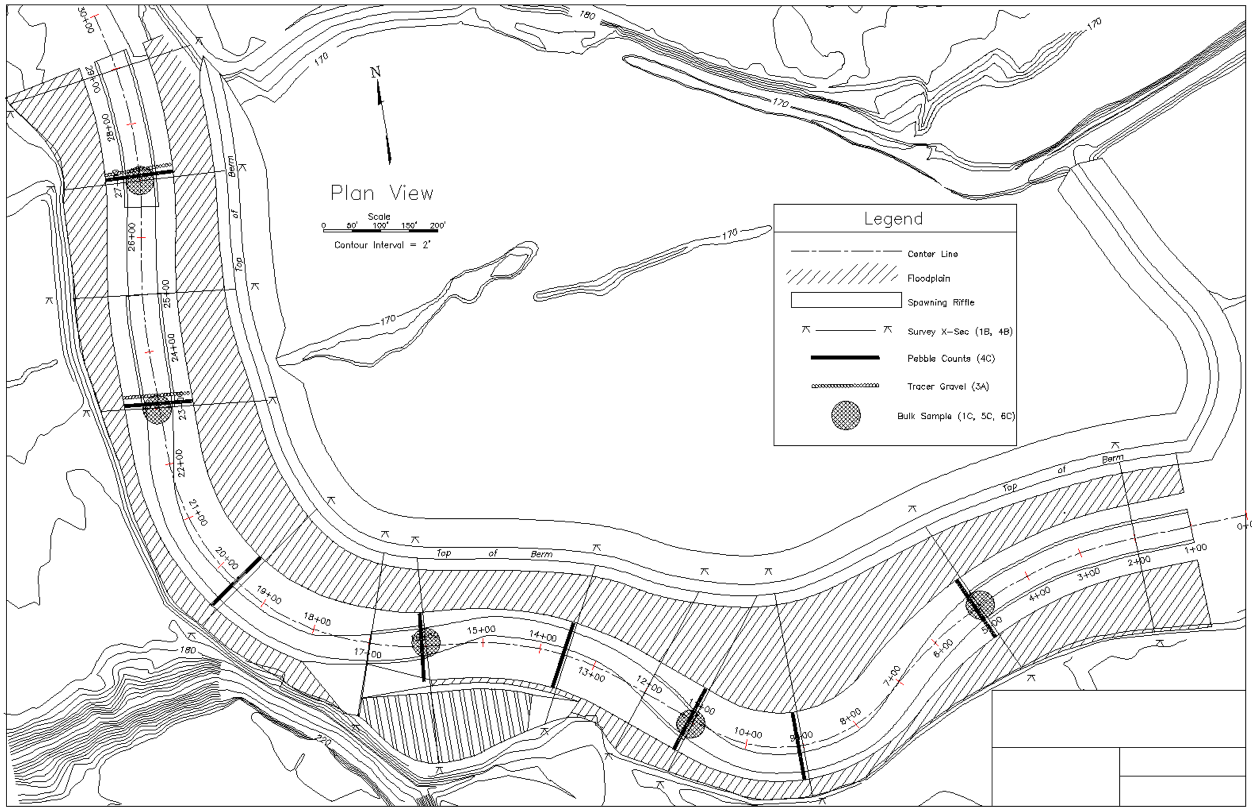

Figure 10 - Monitoring Plan

DWR's monitoring of the project includes several cross-sections at which tracer gravel experiments and pebble counts will be located (Figures 10, 11). These sections and the thalweg profile were surveyed immediately after construction (see as-built drawings, Appendix A). The sections and profile will be surveyed once annually if a flow of greater than 2,000 cfs has occurred, or movement of tracer gravel has been observed. If three consecutive years have not yielded these conditions, a survey of the sections will be completed. The flow of 2,000 cfs was chosen because it is slightly above bankfull, and calculations show that at least 50 percent of the material is mobile at that point. Cross-sections and profiles will be used to document any changes in the storage of alluvium. In addition to the section surveys, a coincident pebble count along with bulk samples will be taken to document any changes in substrate and gravel quality. Thirteen cross-sections will be regularly surveyed - among which eight are designated for the pebble counts and bulk samples. In the baseline monitoring immediately after construction, the pebble counts were completed for all eight sections and bulk samples at five stations. It was determined that in light of the data gathered it was not necessary to take bulk samples on the point bars. This data as well as future monitoring data will be included in a future monitoring report.

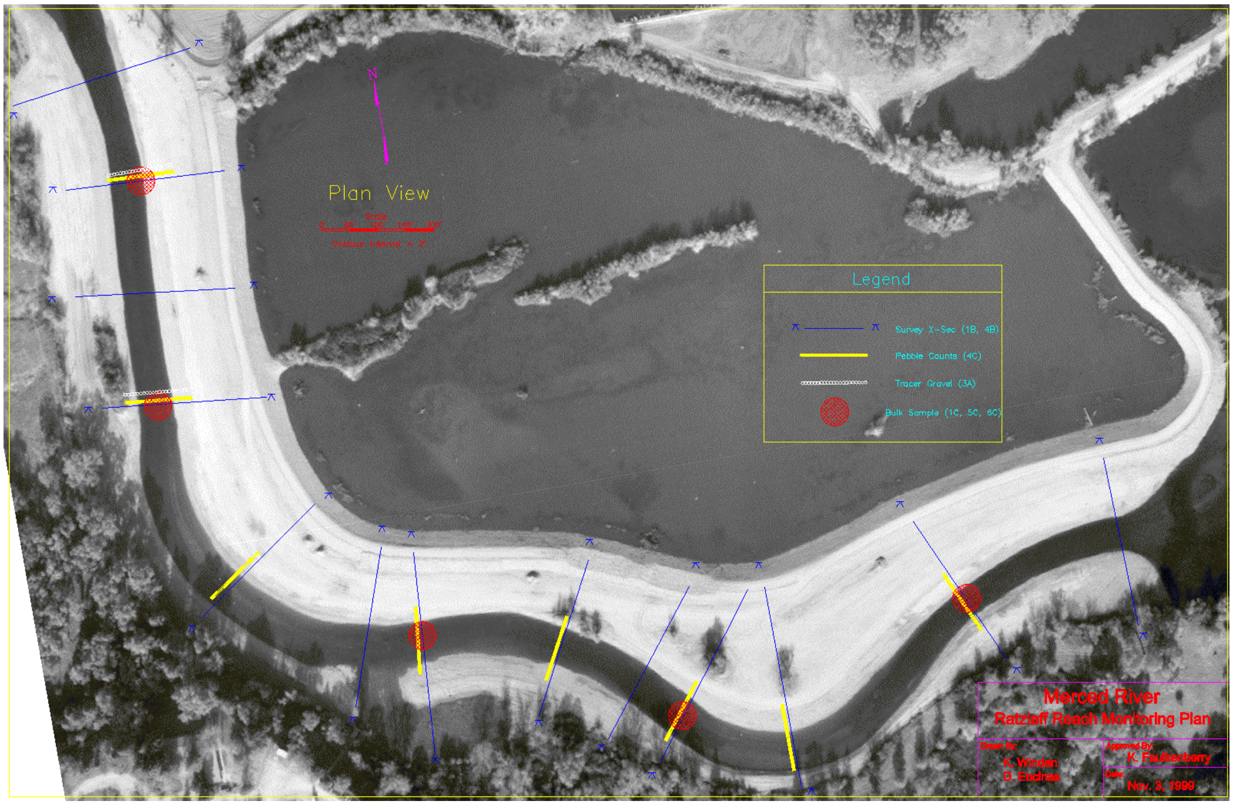

Figure 11 - Monitoring Locations

One area of concern that is being monitored carefully is the “Backwater” area between stations 14+00 and 17+00. This was an area of concern expressed by reviewers of the project plan and will be watched closely using the three monitoring cross-sections at that location.

These monitoring actions, and others to be determined as the project progresses, will allow engineers to assess the effectiveness of the design with respect to the project goals. They will also provide information which will assist in determining volume and location of gravel replenishment projects for the reach in the future.

Conclusion

This project was designed using contemporary methods and techniques, and the goals as outlined in this paper were achieved. The project isolates the predator habitat by separating the river channel from the pond. There will be no direct connection between the two at flows less than 8,000 cfs. Spawning habitat for salmon was created by adding hundreds of feet of riffle to a channel which had none. The goal of improving rearing habitat was achieved by decreasing predator habitat and increasing diversity in the reach by adding riffles, pools, and a backwater area.

The project improves the river dynamics by adding a functional channel and flood plain where there was none. The project was also designed with the current flow regime in mind. The riparian corridor was enhanced by the creation of a flood plain which will be revegetated with native plants according to the revegetation plan. All of these design features will contribute to the creation of a more natural, sustainable stream.

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